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Bridge Design for Multi-Presence of Vehicles

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Bridge Design for Multi-Presence of Vehicles

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Abstract:	It is customary in North American to design or evaluate a short span bridge on a busy highway in exactly the same way as the bridge on a lightly-travelled road. This paper proposes that multi-presence reduction factors can be used as the means for taking account of the volume of traffic. These factors, strictly speaking, are applicable only to those bridges which carry a high volume of traffic; however, in bridge design codes such as AASHTO and OHBDC, thay are currently used for all bridges irrespective of the volume of traffic that they carry. New multiple presence reduction factors, to be used in conjunction with AASHTO, Ontario and CSA codes, are proposed for different classes of highways. In addition, the paper presents simplified methods of transverse load distribution which have the various proposed reduction factors incorporated in them in such a way that the designer or evaluator, while being able to see immediately the effect of the volume of traffic on the live load responses, is not called upon to perform any extra calculations.
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1/ Introduction

It seems intuitively obvious that a bridge on a busy main highway should be designed or evaluated for heavier load combinations than a bridge on a lightly travelled sideroad. In other words, the design, or evaluation loads should depend not only upon the magnitudes of the heaviest vehicle loads, as they currently do, but also upon the density of traffic. Current bridge design codes of North America, namely AASHTO (1984), Ontario (1983) and CSA (1987), do not cater for the density of traffic except when dealing with loading on longer spans which can accommodate more than one vehicle in a lane. This paper provides a basis for making appropriate changes to the loading provisions of these codes so that the density of traffic can be accounted for in a logical and statistically defensible manner in the design or evaluation process. These revised loading provisions can be used with advantage to upgrade, or even remove, load restrictions on structurally suspect bridges on lightly travelled roads.

The proposed changes are made by regarding the current provisions of the various codes as being valid for the busiest highways, and by ensuring that the same probability of load occurrence is maintained for bridges on less busy highways as it is for those on the busiest ones. The net product of this strategy is that the margin of safety, for a given type of bridge, can be maintained at the same level for highways of different traffic densities.

The proposed changes to loading provisions have a considerable effect on the simplified methods of transverse load distribution analysis. This paper also presents the appropriate revisions to those methods of analysis in such a way that the designer, while being able to see the effects of the proposed changes, is not required to perform any extra calculations.

2/ Multiple-Presence Reduction Factors

For short and medium span bridges, the North American codes specify design live loads on a per lane basis, with the loading in a single lane being related to the heaviest vehicles expected to cross the bridge. For multi-lane loading, the design load per lane is reduced to account for the reduced probability of several lanes being loaded simultaneously by heavy vehicles. The reduced "per lane" loading is a scaled-down version of the loading which is specified for single lane loading. To obtain the reduced loading, the single lane loading is multiplied by a factor called the multiple presence reduction factor. Current values of this reduction factor are given in Table 1 for the three North American codes. It is noted that the factors given in this table apply to combinations of static and dynamic loads.

2.1/ Demarcation of Short, Medium and Long Spans

In the absence of generally agreed upon boundaries between short, medium and long spans, the criteria proposed by Bakht et al (1982) can be used to identify the limits. According to these criteria, those bridges in which the maximum load effects are governed by single actual vehicles along the span, can be regarded as short span bridges; those in which the maximum load effects are governed by a train of moving vehicles as medium spans bridges, and those in which the maximum load effects are caused by a train of stationary vehicles with minimum headway distances as long span bridges.

The limit between short and medium spans can be readily established from the dimensions and sizes of actual highway vehicles and assumed headway distances between them. The demarcation between medium and long spans, however, is not deterministic as it depends upon the statistical mix of vehicles. For the work presented in this paper, it is assumed that according to Bakht et al (1982), bridges with spans up to 20 m are short span bridges, and those with spans between 20 and 125 m are medium span bridges. Long span bridges, which clearly have spans larger than 125 m, are not dealt with in this paper.

2.2/ Classes of Highways

The criteria of the Ontario code (1983) can be used to relate the volume of traffic to the class of a highway. According to these criteria, which are given in Table 2, highways are divided into four classes, A, B, C_1 and C_2, with the class A highway being the busiest and class C_2 the most lightly travelled. For purposes of this paper, highways of Class C_1, and C_2 are merged into a single class

which will now be referred to as Class C. It may be noted that Table 2 also provides the criteria for establishing the class of a highway when the traffic counts are not available.

2.3/ The Jaeger and Bakht Method

Jaeger and Bakht (1987) have presented a method for calculating the multi-presence reduction factors along with a proposed set of new factors for the AASHTO and Ontario codes. The various assumptions used in the method and in the development of the proposed factors are discussed in the following.

For a given span, multiple presence is sought only for that fraction of the truck population which represents fully loaded trucks of the type which would prove critical for the span under consideration. It is assumed that, at most, 20% of the population of trucks would fall into this category. On the basis that it will lead to small safe-side errors, the assumption is made that the frequency distribution of the weights of vehicles in this critical category are distributed normally. Following the observation of Csagoly and Knobel (1981), it is assumed that the maximum observed load, $W_{max,1}$ is usually 3.5 standard deviations above the mean, i.e.:

$$W_{max,1} = \mu + 3.5\,\sigma \qquad (1)$$

where μ is the mean weight and σ the standard deviation.

It is further assumed that m heaviest vehicles, each at least of weight $W_{max,m}$, are expected to be simultaneously present in the m lanes of a bridge, and that this maximum vehicle weight is r standard deviations above the mean, so that:

$$W_{max,m} = \mu + r\,\sigma \qquad (2)$$

The multi-presence reduction factor for static loads, which is denoted as m_{fs}, is defined as the ratio of $W_{max,m}$ to $W_{max,1}$:

$$m_{fs} = W_{max,m} / W_{max,1} \qquad (3)$$

The multiple presence is investigated in the middle one-third spans of short span bridges with m loaded lanes, in which the traffic flow in different lanes are unrelated.

By using Poisson's parameter, Jaeger and Bakht (1987) have derived expressions for probability of simultaneous presence of vehicles; they have shown that the values of both m_{fs} and m_f, the latter being the reduction factor for combined static and dynamic loads, depend upon the factor BT^{m-1}, where B is the bridge life in years, T is the average time in seconds taken by a vehicle to transverse the middle one-third of a span and m is the number of loaded lanes. Plots of m_f against BT^{n-1} are reproduced from Jaeger and Bakht (1987) in Fig. 1 for different classes of highways and number of loaded lanes.

The significant outcome of the work of Jaeger and Bakht (1987) is not the set of multiple-presence reduction factors, that they proposed, but the procedure for calculating the probabilities of multiple presence. The proposed reduction factors can always be open to question, as their values depend upon both the assumptions made in the derivation of the equations and subjective estimates of the Poisson's parameter. However, if the value of a reduction factor is given for a certain traffic density, the procedure of Jaeger and Bakht can be used with advantage to calculate the corresponding values of the reduction factor for different densities of traffic, in such a way that the new factors correspond to the same probability of occurrences as the given factor.

2.4/ Proposed Factors

By treating the current AASHTO multiple presence reduction factors as being valid for class A highways, the corresponding factors were calculated for class B and C highways by the Jaeger and Bakht (1987) method using the above approach. Conservatively, the resulting factors were rounded upwards to the nearest 5th percentile. The current reduction factors for Class A highways and the new factors for Class B and C highways, which are proposed to be used in conjunction with the AASHTO specifications (1984), are listed in Table 3. It can be seen in this table that for certain reduction factors, asterisks are given instead of numerical values; these factors correspond to a condition in which the multi-presence of trucks of any weight is an unlikely event. For example, an asterisk is given in Table 3 for the reduction factor corresponding to the four-lane loading condition on a class C highway. This conforms to engineering judgement, following which it can be foreseen that a bridge on a lightly travelled highway is unlikely ever to experience four side-by-side trucks of any weight on the middle one-third of its spans.

As can be seen in Table 1, the current multiple presence reduction factor for the draft CSA code (1987) are the same as for the Ontario code (1983). Accordingly, both these codes can use the same set of proposed reduction factors which are given in Table 4 and which were calculated by the same procedure as that used for the proposed factors for AASHTO specifications.

The proposed Ontario and CSA reduction factors can be seen to be somewhat smaller than the respective AASHTO factors. This is because, following the work of Harman and Davenport (1976), the reduction factors for the Ontario code were made smaller than the AASHTO factors. The work of Jaeger and Bakht (1987) confirms that the Ontario reduction factors better represent the actual situation than the AASHTO factors.

3/ Transverse Load Distribution

The North American bridge design codes require that when more than one design lane is loaded, the design loading, which is specified for a single lane, should be multiplied by the appropriate multiple presence reduction factor; and the design lanes to be loaded should be selected so as to maximize the load effects. This requirement, clearly, necessitates a number of transverse load distribution analyses and the use of the reduction factors. Yet, most engineers in North America rarely, if ever, use the reduction factors directly, because these factors and the search for the maximum load effects are implicit in the simplified methods of analysis that these engineers commonly use for bridge design and evaluation. A paper on multiple-presence reduction factors can, therefore, be of use to the practicing engineer only if it also deals with the effect of the revision of these factors on the simplified methods of transverse load distribution analysis. Accordingly, to make this paper immediately useful to the practicing bridge engineer, simplified methods of load distribution analysis are presented in the following to deal directly with the different multiple-presence reduction factors.

3.1/ Current Simplified Methods

The simplified methods of transverse load distribution analysis specified in the North American bridge design codes reduce the complex analysis of a bridge subjected to one or more design vehicles to the simple analysis of a beam. With these methods, the maximum load effects in a girder, or in a strip of unit width in the case of a slab bridge, can be obtained by treating the girder, or the strip of unit width in the case of a slab bridge, as a one-dimensional beam subjected to a loading which is obtained by multiplying one line of wheels of the design vehicle by a load fraction (S/D). In this approach, S is the girder spacing, or a unit width in the case of a slab bridge, and the quantity D, which has units of length and which is a measure of the transverse load distribution characteristics of a bridge, is specified directly in a given code, and varies from one code to another.

AASHTO specifications (1984) provide the simplest means of obtaining the values of D; they provide a single value of D for each type of bridge, and they require that the same value of D be used for obtaining both the maximum moments and shears in the girders or strips of unit width. Arising from its extreme simplicity, the AASHTO method of analysis has been found to be unrealistic for at least some bridges.

The Ontario method (1983), while still being simple, requires that D be obtained by a modest amount of calculation; according to this method, the design value of D, D_d, is obtained from the following expression for obtaining longitudinal moments.

$$D_d = D\{1 + \mu c_f/100\} \quad (4)$$

where: $\mu = (W_e - 11)/2$ for U.S. customary units

$\mu = (W_e - 3.3)/0.6$ for metric units (5)

W_e is the design lane width in feet for U.S. customary units and in meters for metric units. The values for D and C_f are read from graphical charts which use two dimensionless characterizing parameters as axes; these parameters are calculated from the bridge geometry and properties of its cross-section. For longitudinal shear, the value of D can be read directly from tables, depending upon the type of the bridge and the number of design lanes in it. For longitudinal moments the Ontario method provides D values separately for internal and external girders (or strips of unit width).

Bakht and Jaeger (1985) have provided a similar method which can be used in conjunction with the AASHTO specifications. It is noted that the Ontario method, like the simplified AASHTO method (1984), was developed from the results of a large number of rigorous analyses in which the load effects in the bridges were maximized by loading them successively in different lanes. Since the relevant multi-presence reduction factors were applied to the loads in the process of maximizing the load effects, these factors are implicit in the D values and are "transparent" to the user of the method.

The CSA method (1987) of obtaining D values was derived from the results of the same analyses which led to the Ontario method; however, it is similar in format to the AASHTO method except that separate values are provided for moments and shear, and that these values can be read directly from tables depending upon the bridge type and number of lanes in it.

3.2/ Bakht and Moses Method

A further simplification in obtaining the D value for moments in slab-on-girder and slab type of bridges has recently been introduced by Bakht and Moses (1988); they have shown that values of D can be expressed with good accuracy as functions of the span length. For slab-on-girder

bridges, the simplification was made possible by empirical relationships between span lengths and the longitudinal flexural rigidity per unit width of typical slab-on-girder bridges in North America. For slabs, and similar, bridges, advantage was taken of the fact that for these bridges one of the characterizing parameters has an almost constant value, and the other parameter already has a direct relationship with the span length without involving the various rigidities of the bridge.

The Bakht and Moses method (1988) is similar in format to the Ontario method, i.e., it makes use of Eqs. 4 and 5. The values of C_f and D for both external and internal girders (and strips of unit width in the case of slab bridges) are provided against the axis of span length. The charts of C_f and D are different for the AASHTO codes, as these codes specify different sets of multiple presence reduction factors.

3.2/ Revision of Bakht and Moses Method

As mentioned earlier, Bakht and Moses (1988) used the results of the same analyses which were conducted to develop the Ontario simplified method of transverse load distribution analysis. The same data were used again to revise the Bakht and Moses method so that the new sets of reduction factors could be incorporated in it. The resulting set of charts are given in Figs. 2, 3 and 4 for two-, three- and four-lane slab-on-girder bridges, respectively. As can be seen in these figures different charts are provided for external and internal girders for each of the AASHTO specifications and the Ontario Highway Bridge Design Code (OHBDC). The charts for the latter are also applicable to the CSA code. The same charts for C_f are applicable to all three codes.

With the help of the charts given in Figs. 2 through 4, the values of D and C_f for a slab-on-girder bridge can be read directly against the span length corresponding to any of the highway classes. The design value of D, i.e. D_d, can then be easily obtained from Eqs. 4 and 5. The charts of Figs. 2 through 4 also provide a ready means of seeing the effect of the highway class on live load moments. For example, it can be seen that for a two-lane slab-on-girder bridge with a span of 30 m, the AASHTO D values for internal girders are 1.73 and 1.93 m for highway classes A & C, respectively. If the design lanes are each 3.3 m wide, the change of highway class from A to C would correspond to a reduction of live load moments of about 12%. Such a considerable reduction of moments can be used with advantage in the evaluation of marginally deficient bridges on low volume roads.

The charts for moment D and C_f values for two-, three- and four-lane slab and similar types of bridges are given in Figs. 5, 6 and 7, respectively. These and the other charts have been labelled sufficiently for them to be self-explanatory.

3.3/ D values for Shears

Following Ontario (1983) and CSA (1987) codes and the recommendation of Bakht and Jaeger (1985), a single value of D for shear can be used realistically for a particular type of bridge with a given number of design lanes. Using the same basic data which was used to develop the shear D values given by Bakht and Jaeger (1985), new values of D were calculated for class B and C highways using the proposed multi-presence reduction factors given in Tables 3 and 4. The resulting values of D for shear, which can be used in conjunction with the AASHTO specifications, are given in Table 5, and those which can be used in conjunction with both the Ontario and CSA codes are given in Table 6.

It is noted that in the case of shear, no correction of the form represented by Eq. 4, is required in order to obtain the design values for D. The values given in Tables 5 and 6 can themselves be regarded as the final values.

4/ Dealing with Traffic Volume Through Live Load Factors

A seemingly attractive method of accounting for the volume of traffic in the load capacity evaluation of existing bridges would be through the use of live load factors. The method would require that the live load factor have a prescribed smallest value when the traffic volume is the lightest and the law enforcement the most rigorous. Conversely, when the traffic volume is most dense and the law enforcement almost non-existent, the live load factor has its highest prescribed value. Despite its attractiveness, this approach must be questioned on the basis that, as shown in the following, it can lead to different design loads, and hence different safety levels, in bridges carrying the same volume of traffic.

It is well established that with a given degree of law enforcement, the magnitude of the weight of the heaviest vehicle is not much affected by the volume of traffic.. It is, therefore, reasonable to expect that the maximum single lane loading on a short span would be the same for all classes of highways. Now, consider the example of a two-lane bridge with four girders, which has extremely poor transverse load distribution characteristics so that the directly loaded girders take virtually all the loads themselves, without passing any significant amount onto the other girders. When this bridge carries a single vehicle of weight W in the manner shown in Fig. 8(a), the moments at any section in the two girders not carrying the loads directly, are very nearly zero. This bridge is now subjected to two side-by-side vehicles, each weighing $f_s w$, with f_s being the multiple presence reduction factor which is less than 1.0. As shown in Fig. 8 (a), the girder moments due to the latter load case would always be less than those due to the former, so that the governing loading condition for the bridge under consideration is that with loading in only one lane.

Since the magnitude of single lane loading is not affected by the volume of traffic, it can be seen that for the bridge under consideration, the net load effects would remain the same irrespective of the highway class. For such a bridge on a low volume road, a lower live load factor would necessarily reduce the margin of safety.

If the transverse load distribution characteristics of the bridge are improved considerably by making the deck slab thicker and introducing a large number of transverse diaphragms, then as shown in Fig. 8(b), the single lane loaded case may no longer remain the governing one. In this case, the changing of the live load factors may be appropriate. However, it is noted that the live load factors should be functions of loads; when they also depend upon the load distribution characteristics of bridges, their validity becomes open to question.

It is strongly endorsed that the concept of making the live load factor depend upon the degree of law enforcement is not only valid but is also extremely useful.

5/ A Note About the Basic Assumption

The proposed multiple presence reduction factors were developed for short span bridges. It may not be reasonable to assume that these factors can also represent accurately the reduction of loading in medium span bridges which comprises more than one vehicle in a lane. However, the use of the same reduction factor, for medium span bridges is accepted on the ground that it leads to a considerable safe-side error, part of which is expected to be compensated by non-random grouping of vehicles, the latter not having been considered in the calculation of the probabilities.

6/ Summary of Recommendations

Using the highway class system defined in Table 2 as standard, the following recommendations are made.

1. The respective multiple presence reduction factors currently specified in the AASHTO (1984), Ontario (1983) and CSA (1987) codes should be regarded as valid for class A highways.

2. The modification factors for class B and C highways given in Table 3 should be adopted by the AASHTO specifications, and those given in Table 4 should be adopted by the Ontario and CSA codes.

3. For simplified analysis of transverse load distribution, the extension of the Bakht and Moses method (1988) presented in this paper is recommended to be used. According to this method, the values of D for moments corresponding to any of the three classes of highways can be read directly against the span length from the relevant one of the charts given in Figs. 2 through 7. The D values for shear can be read from the relevant of Tables 5 and 6.

7/ Conclusions

It has been demonstrated that in the calculations for bridge design or evaluation account can be taken of the volume of traffic through multiple presence reduction factors. The paper not only proposes new reduction factors for different classes of highways, but also presents simplified methods of analysis which incorporate these reduction factors. The net result of the proposed procedure is that the designer or evaluator can readily ascertain the effect of the volume of traffic in the main live load responses.

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Table 1/ Current Multiple-Presence Reduction Factors

Code	Reduction factor for load in... 1 lane	2 lanes	3 lanes	4 lanes	5 lanes	6 or more lanes
AASHTO (1984)	1.00	1.00	0.90	0.75	0.75	0.75
Ontario (1983)	1.00	0.90	0.80	0.70	0.60	0.55
CSA (1987)	1.00	0.90	0.80	0.70	0.60	0.55

Table 2/ Definitions of highway classes according to the volume of traffic — Ontario Code (1983)

Highway class	Average No. of trucks: per lane per day	Average No. of all vehicles: per lane per day	Criteria when traffic data are not available
A	>1000	>4000	Roads primarily for through traffic
B	>250 and <1000	>1000 and <4000	Roads on which traffic movement and property access have similar importance
C_1	>50 and <250	<100 and <1000	Roads primarily for property access which carry moderate commercial traffic
C_2	<50	<100	Roads primarily for property access which carry little or no commercial traffic

Note: Classes C_1 and C_2 comprise, together, the single Class C used in this paper.

Table 3/ Proposed multiple-presence reduction factors to be used in conjunction with AASHTO specifications

Highway class	Average No. of trucks per lane per day	No. of loaded lanes 2	3	4	5 or more
A	>1000	1.00	0.90	0.75	0.75
B	>250 and <1000	0.95	0.85	0.70	*
C	>50 and <250	0.90	0.75	*	*

**These cases need not be considered at all*

Table 4/ Proposed multiple-presence reduction factors to be used in conjunction with Ontario and CSA codes

Highway class	No. of loaded lanes 2	3	4	5	6 or more
A	0.90	0.80	0.70	0.60	0.55
B	0.90	0.80	0.70	*	*
C	0.85	0.70	*	*	*

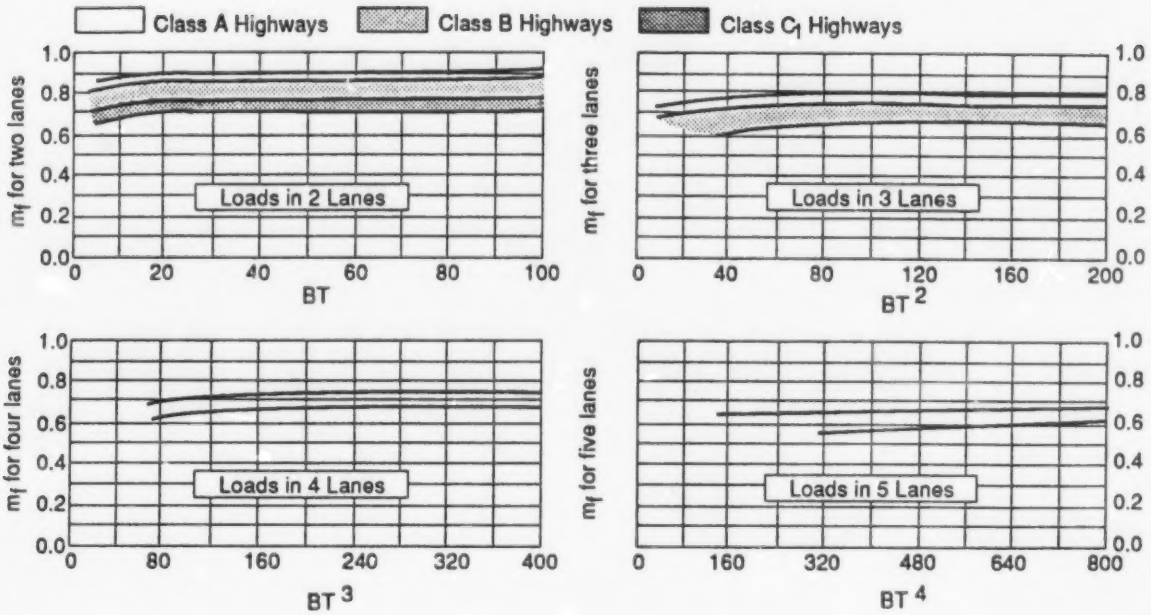
**These cases need not be considered at all.*

Table 5/ Shear D values in metres to be used in conjunction with AASHTO specifications

Bridge type	Hwy. class	No. of lanes in the bridge		
		2	3	4
Slab-on-girder	A	1.60	1.70	1.90
	B	1.70	1.80	2.00
	C	1.80	1.90	2.10
Slab and voided slab	A	1.95	2.00	2.15
	B	2.00	2.10	2.25
	C	2.10	2.20	2.40

Table 6/ Shear D values in metres to be used in conjunction with Ontario and CSA codes

Bridge type	Hwy. class	No. of lanes in the bridge		
		2	3	4
Slab-on-girder	A	1.70	1.85	1.90
	B	1.70	1.85	1.90
	C	1.80	1.90	2.10
Slab and voided slab	A	1.95	1.95	2.15
	B	1.95	1.95	2.15
	C	2.05	2.05	2.30



Notes: 1. For class B highways, loading in more than 3 lanes need not be considered at all
2. For class C highways, loading in more than 2 lanes need not be considered at all

3. B = bridge life in years; T = time interval in seconds required for a vehicle to cross the middle one-third span under consideration

Figure 1/Multiple presence reduction factors
(Jaeger and Bakht, 1987)

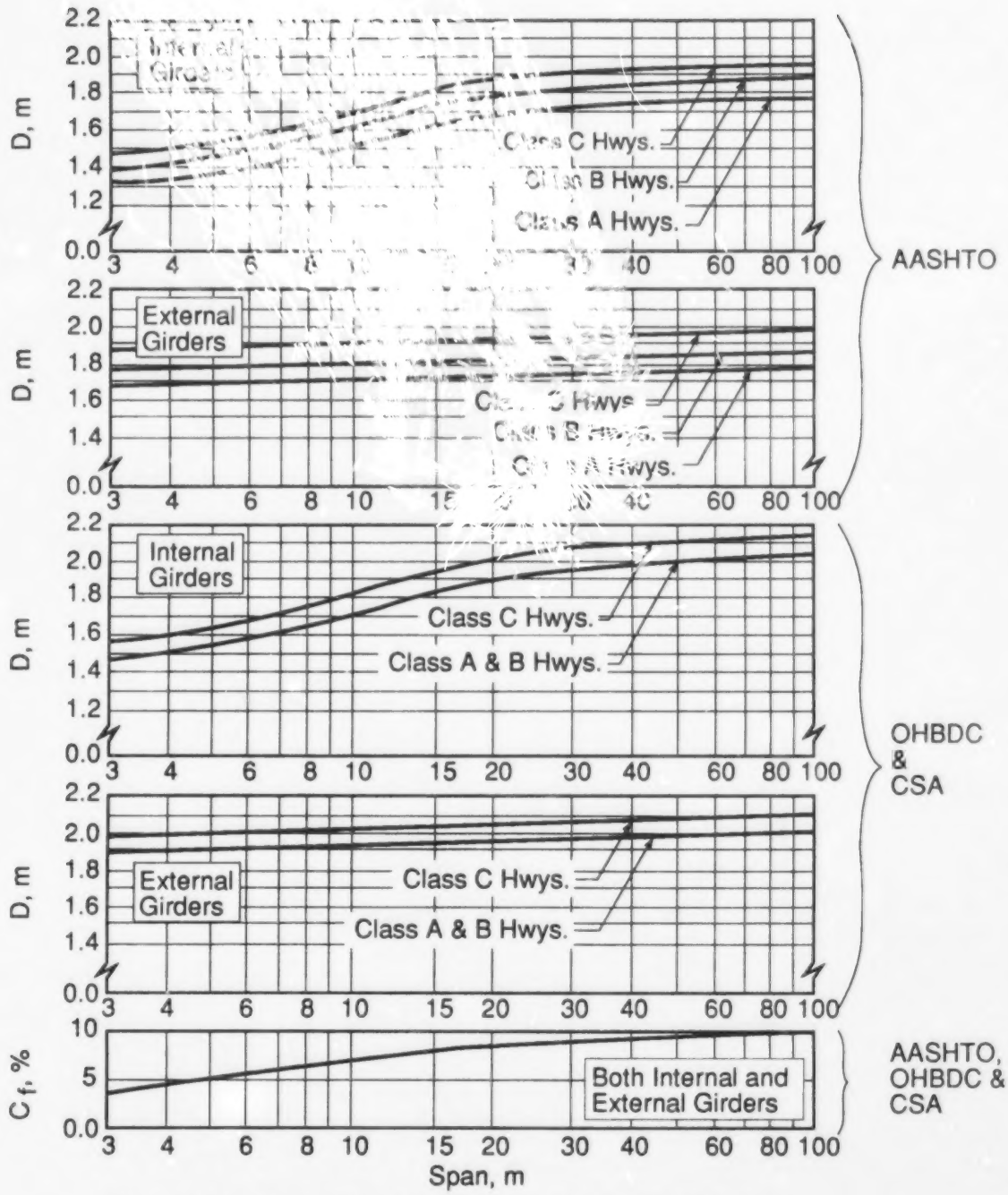


Figure 2/ D and C_f values for 2-lane slab-on-girder bridges

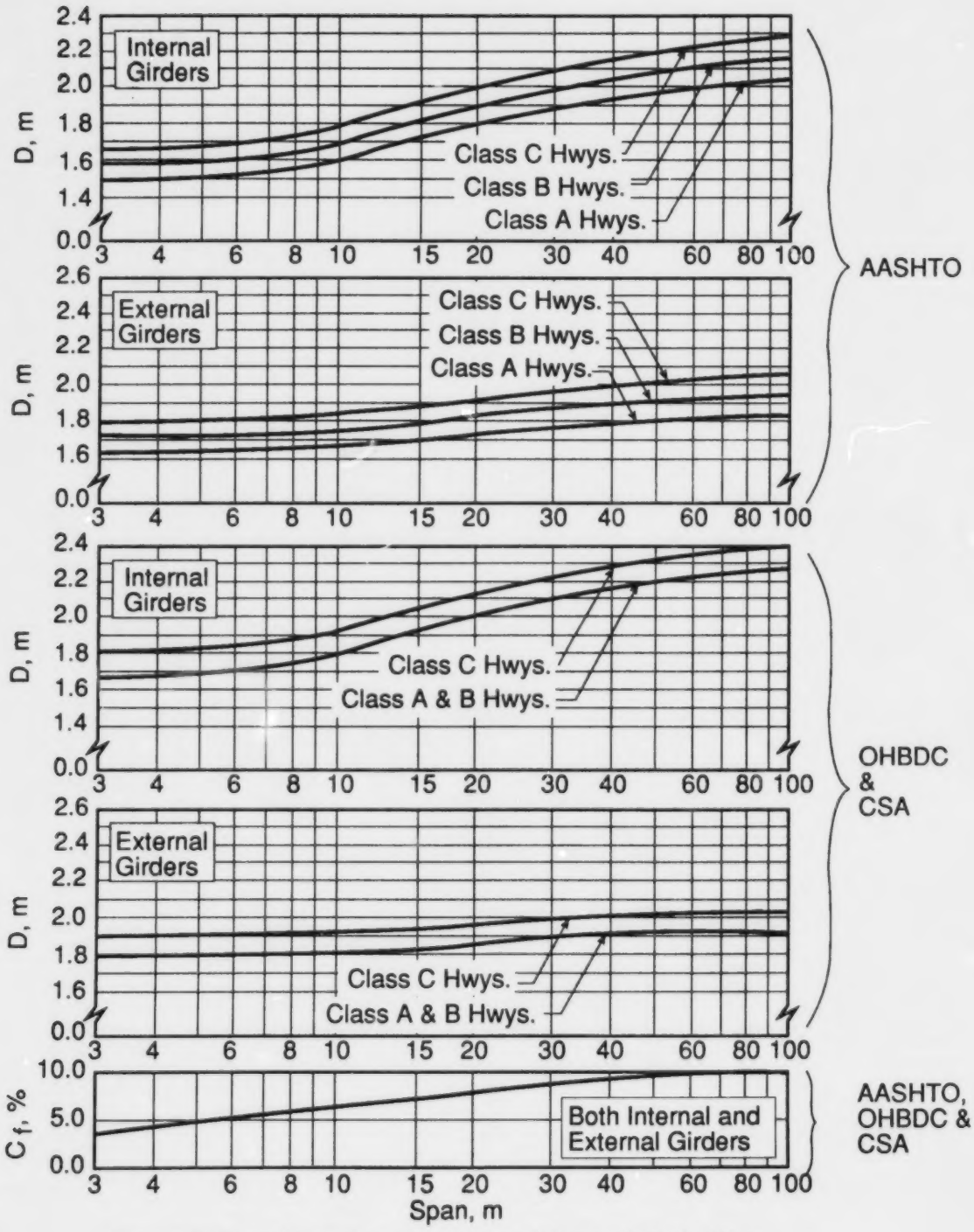


Figure 3/ D and C_f values for 3-lane slab-on-girder bridges

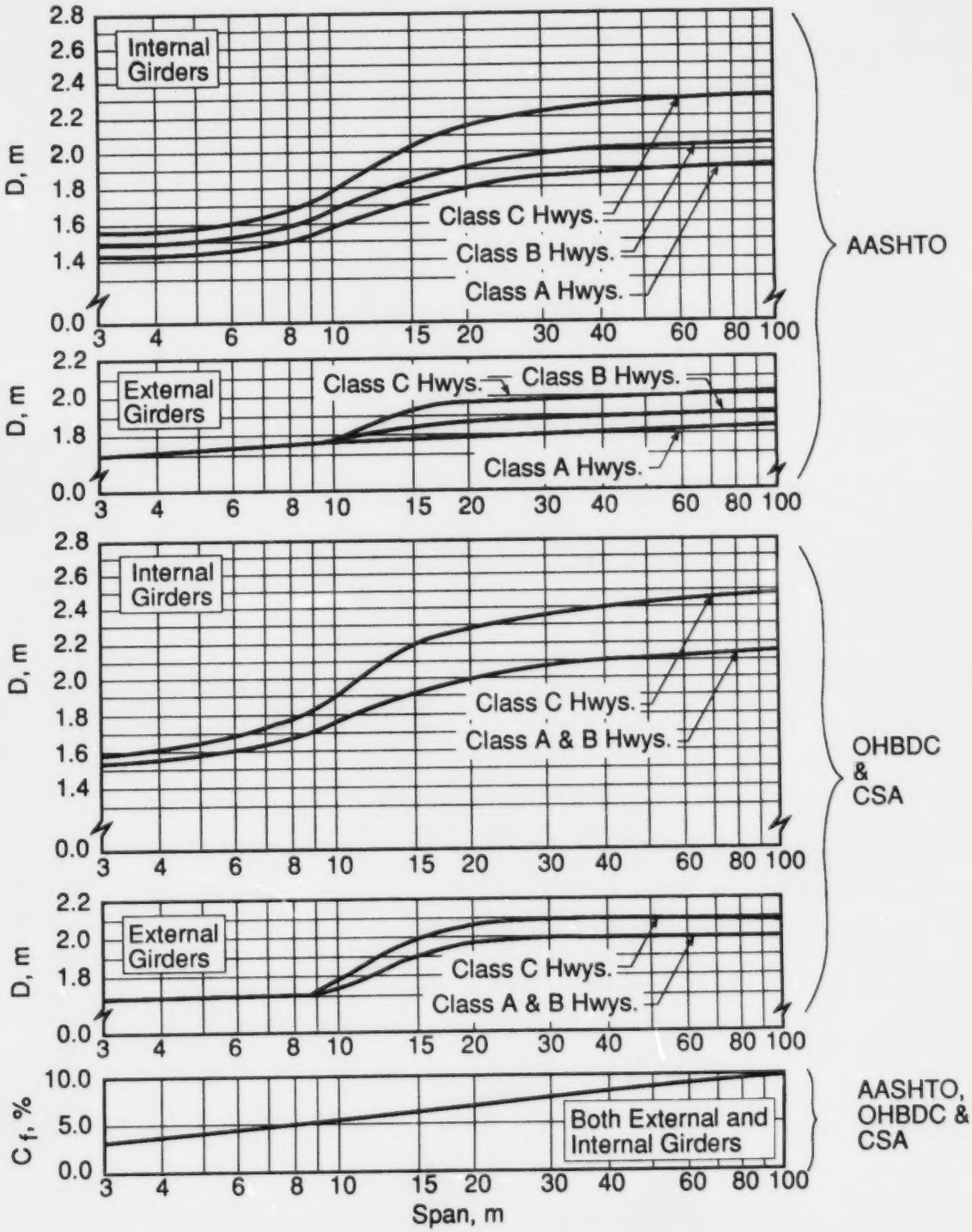


Figure 4/ D and C_f values for 4-lane slab-on-girder bridges

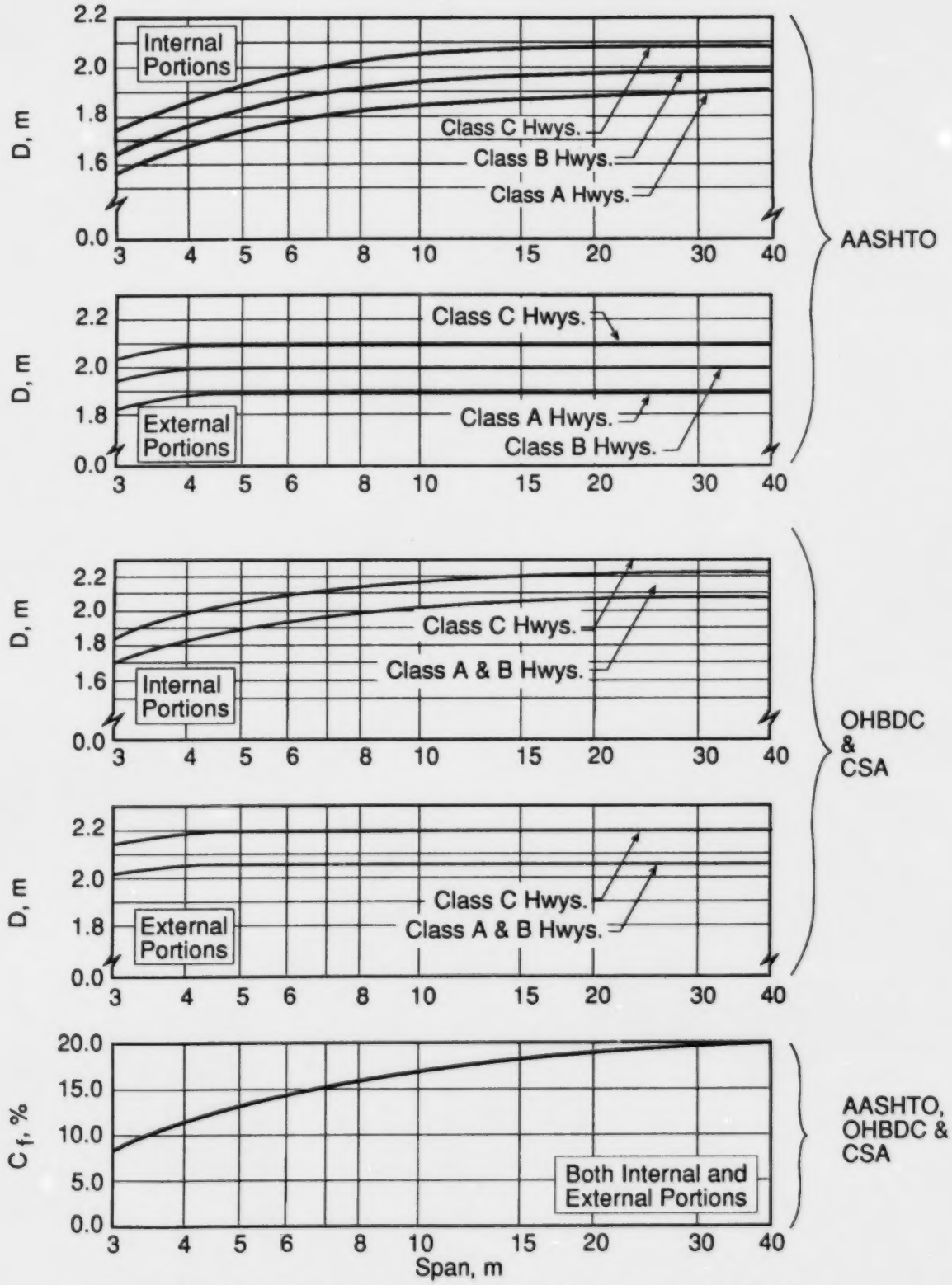


Figure 5/ D and C_f values for 2-lane slab bridges and voided slab bridges

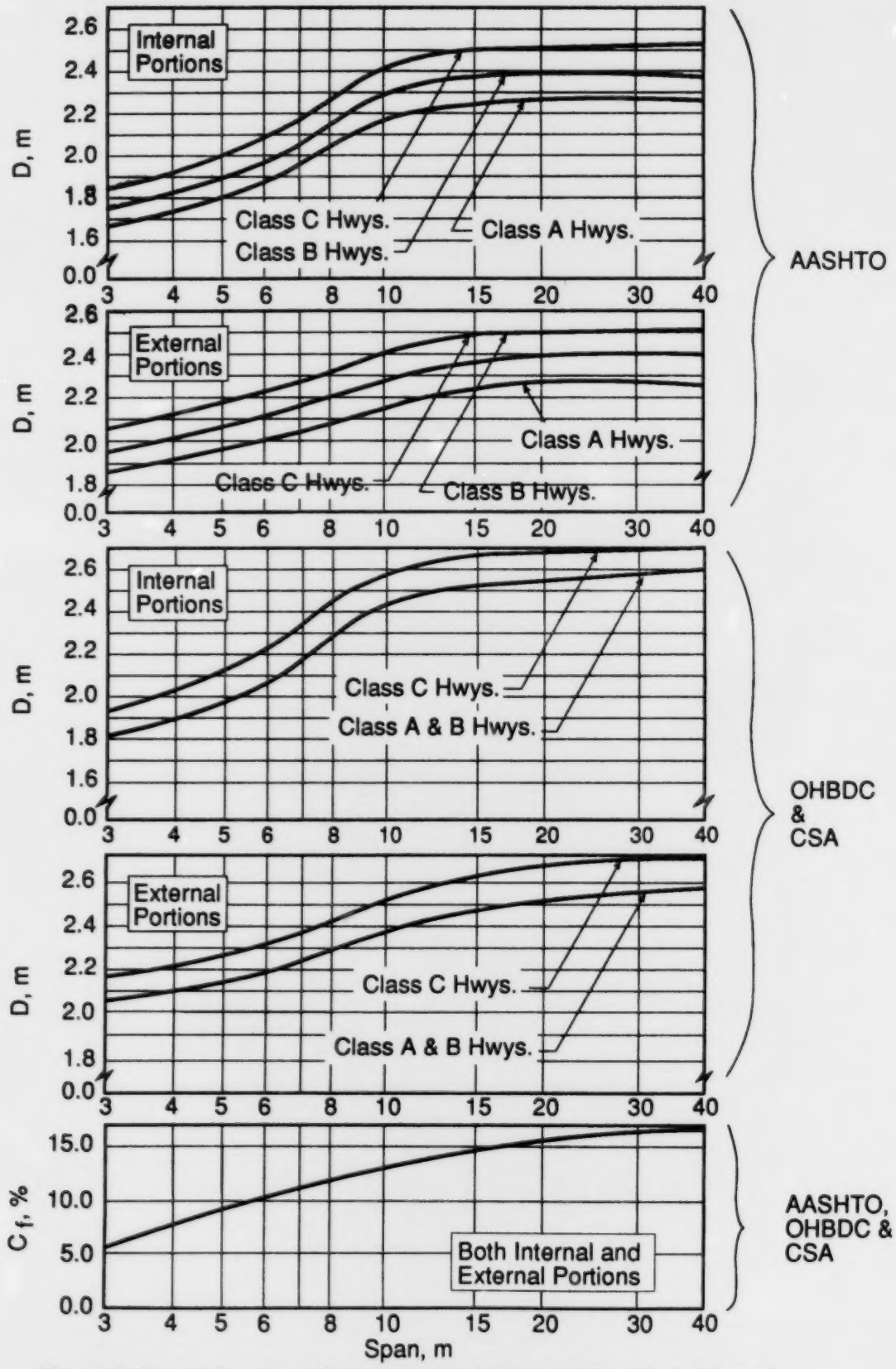


Figure 6/ D and C_f values for 3-lane slab bridges and voided slab bridges

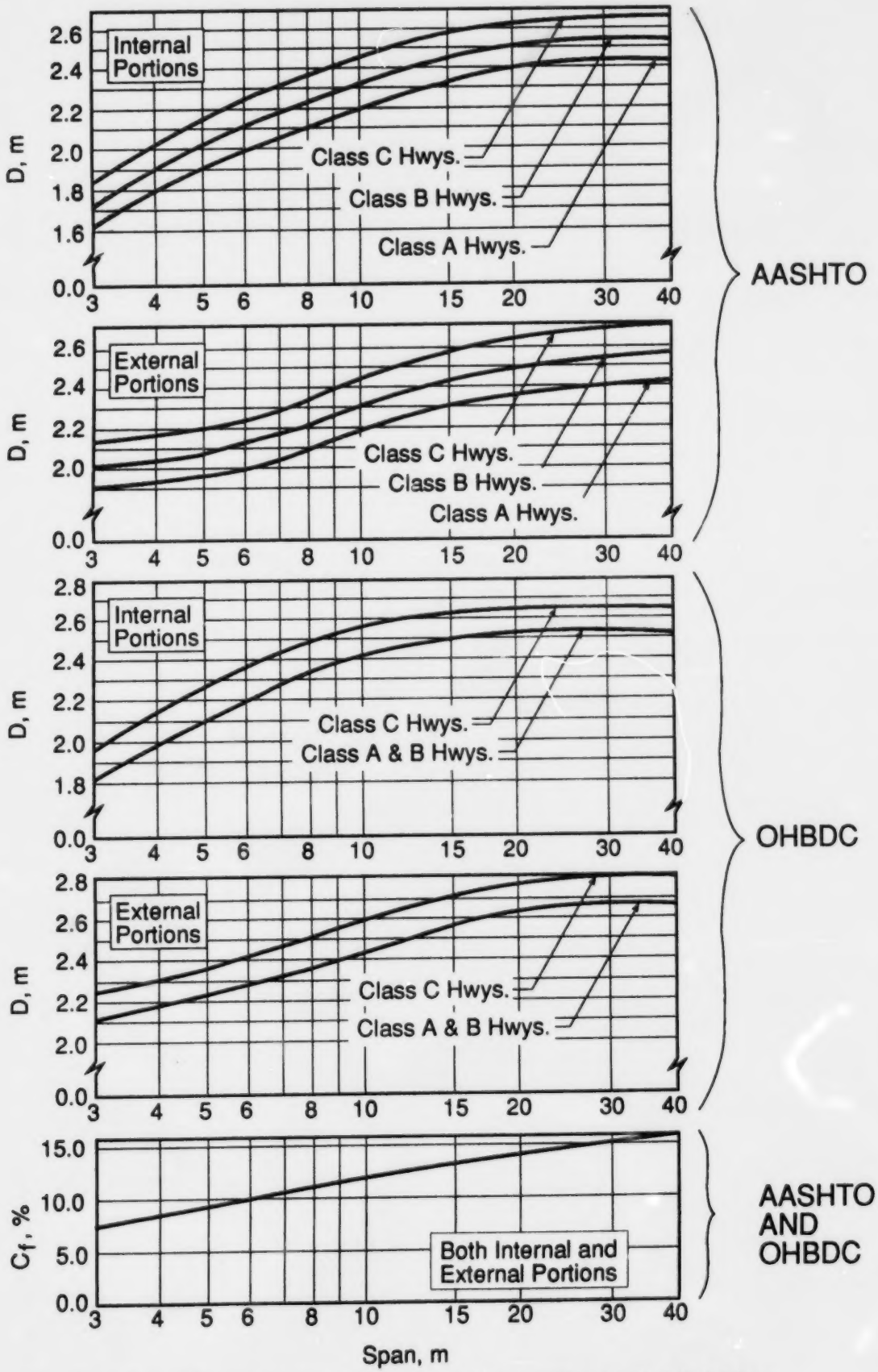
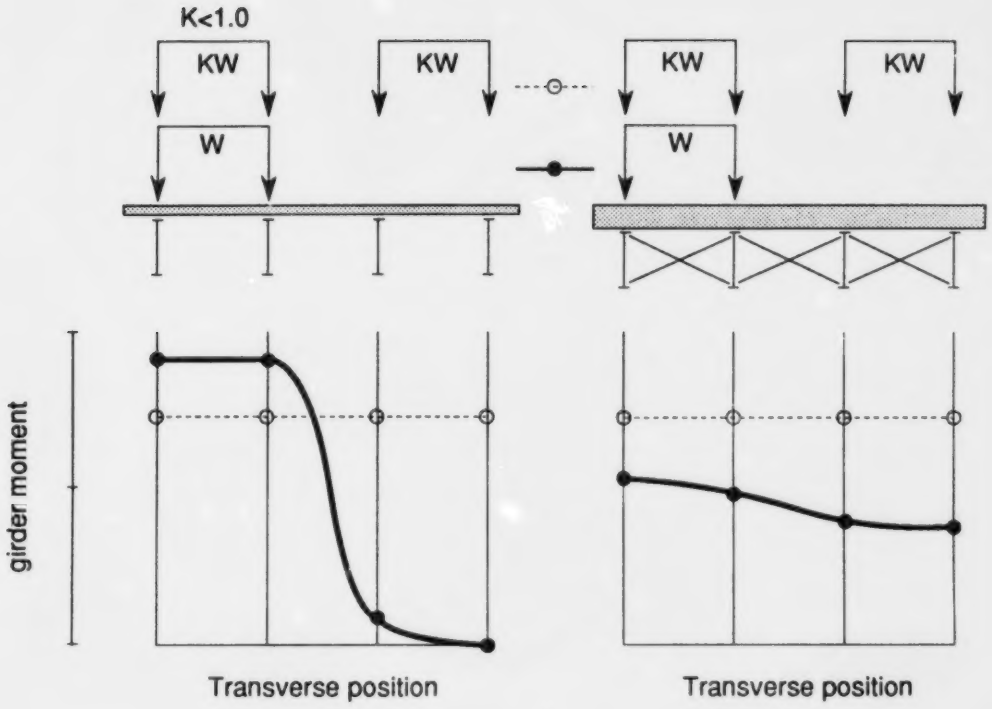


Figure 7/ D and C_f values for 4-lane slab bridges and voided slab bridges



(a) bridge with very poor load distribution characteristics

(b) bridge with very good load distribution characteristics

Figure 8/ Girder moments in two bridges